Nonlinear Seismic Analysis and Retrofit of BC Place Stadium Using Rocking Foundation and Viscous Dampers

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SUMMARY

BC Place Stadium located in Vancouver, Canada, has gone through an extensive structural upgrade including the gravity elements of the stadium supporting the new roof as well as a seismic upgrade of the stadium's lateral load resisting system. The existing concrete frame structure constructed in the early 1980's was divided into eight segments and eight periphery ramp structures by expansion joints. One of the major upgrades to the stadium was the demolition of the existing air supported fabric roof and its replacement with a structurally supported roof. The new roof structure consists of a partial fixed and partial retractable fabric roof canopy supported by cables and structural steel pylons around the perimeter of the existing stadium ring beam.

The existing lateral load resisting system of the stadium (concrete bowl structure) is comprised of a series of circumferential shear walls located in each of the eight stadium segments and 54 concrete frame structures in the radial direction with closely spaced reinforcing ties placed in the columns and joints. Circumferential concrete beams connect the radial frames laterally to the circumferential shear walls.

To minimize the cost of seismic upgrading of the existing concrete bowl structure the design team carried out the base-motion time history analysis focused on rocking foundation and nominal yielding of some of the beams. As part of further improvements to the lateral load resisting system, 96 viscous dampers were also installed in the stadium at existing expansion joints between bowl segments and ramp structures to absorb seismic energy. This helped to optimize the seismic upgrading of diaphragms, walls, frames and foundations.

This paper will present an overall review of the stadium's gravity and lateral load resisting system, how and why the design team introduced nonlinear response history analysis including rocking foundation and added viscous damping to the structure.

Keywords: Rocking Foundation, Viscous Damper, Retrofit

1. INTRODUCTION

An iconic building for the City of Vancouver, BC Place stadium is located at 777 Pacific Boulevard on the North side of False Creek. The construction of this 60,000 seat stadium commenced in April of 1981 and completed in June of 1983. The stadium has recently undergone interior renovations including several structural upgrades to accommodate a new roof structure. Elements of the existing structure that resist seismic forces from the new roof have been upgraded to meet the current seismic design provisions (e.g. upgrades have been designed in conjunction with the new roof).

When designed, the governing design statute was the National Building Code of Canada, 1980. Seismic analysis of the structure consisted of a static force method, as prescribed by the building code, and these forces were applied to two-dimensional computer models as well as hand calculations. Detailing of the concrete reinforcement was completed to a high standard. Subsequent editions of the building codes have increased the magnitude of earthquake for which a designer must consider in the design. Risk involving earthquakes are commonly expressed as a probability of exceedance over a given time period. As shown in Table 1 below, the period of exceedance has increased, which has resulted in an increase in level of seismic forces that must be accommodated by a new structure's lateral load resisting system. Beyond the increase in return period, an importance factor has been incorporated for buildings likely to be used as post-disaster shelters in current BC building code; this additional factor also increases the design forces.

Table 1: Seismic Base Shears of Various Building Codes

Design Statute	Seismic (Elastic) Base Shear	Return Period
NBCC 1980*	$V_E = 0.14W$**	1/100
NBCC 1995*	$V_E = 0.33W$	1/475
NBCC 2005***	$V_E = 0.66W$***	1/2475

*Importance Factor, I_E, of 1.0
**Original seismic design criteria
***Importance Factor (I_E) taken as 1.3 and assuming fixed base shear walls

The seismic assessment of BC Place stadium involved numerous analytical models/methods with emphasis on the soil-structure interaction, as discussed below.

2. STRUCTURAL SYSTEM

BC Place Stadium is a concrete structure that is comprised principally of 54 concrete moment frames orientated radially that support four suspended levels of precast concrete joists and bleacher sections which form the floors of the elevated structure. The weight of the structure is approximately 690,000 kN (155,126 kips) without the addition of the new roof and 800,000 kN (179,856 kips) inclusive of the new roof structure. The eight periphery ramp structures share common footings with the existing stadium shear walls on Gridline F and add a total of 220,000 kN (49,460 kips) of additional weight. The overall plan dimensions of the building are 224m by 183m, with the bowl frames arranged as a super ellipse. The top of concrete event level slab is 3m above the sea level. The top of the existing structure is approximately 35m above the event level; with the new roof in place the building will project approximately 82m above the event level (See Fig. 1).

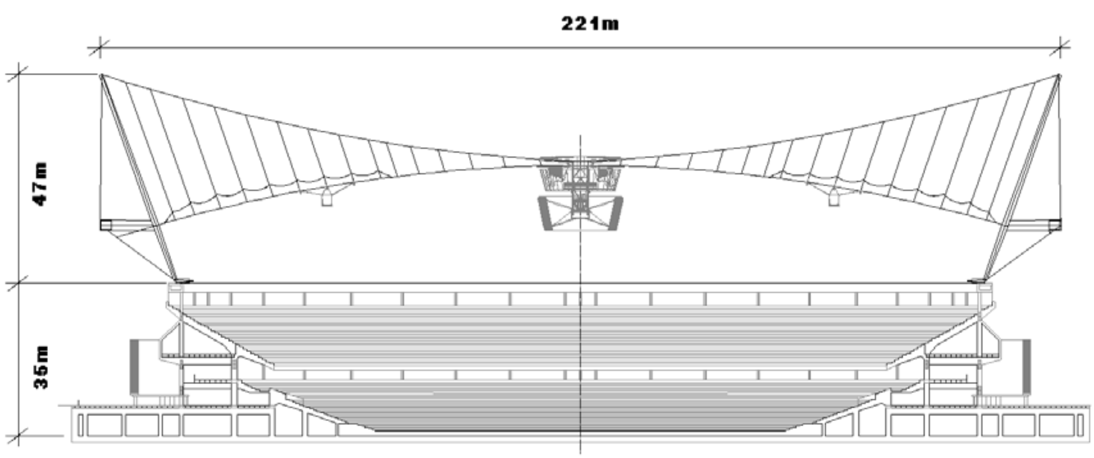

Figure 1. An overall transverse section of BC Place existing bowl and exterior plaza with the new roof (original drawing courtesy of Geiger Engineers)

The radial moment frames act to support the gravity loads imparted by the self-weight of the structure, occupancy, and snow loads. In addition, these frames provide lateral stiffness to resist wind and seismic effects. Detailing of the steel reinforcement in the concrete frames was found to be generally acceptable although column upgrades at Gridline F were still required to support the new roof structure.

The original design also includes 42 concrete shear walls orientated circumferentially throughout the stadium (See Fig. 2). These walls are located on all five circumferential grid lines and vary in height from the underside of roof level at the perimeter of the building to the underside of precast joists below the Level 3 suites in the concourse area, and to the underside of the bleachers in the lower bowl. The orientation of these shear walls is intended to resist lateral forces that would otherwise burden the concrete moment frames in a direction perpendicular to their orientation, or the “weak axis” of the moment frames. For reasons of thermal expansion and concrete creep effects, the building was partitioned into eight segments, with expansion joints at Gridlines 2, 9, 16, 23, 29, 36, 43, and 50 (See Fig. 2). The ramps are each laterally supported by two radial shear walls as well as one wall parallel to the existing stadium shear walls outboard of Gridline F (See Fig. 2); these walls are not connected to the bowl structure (generally with a gap of at least 60 mm).

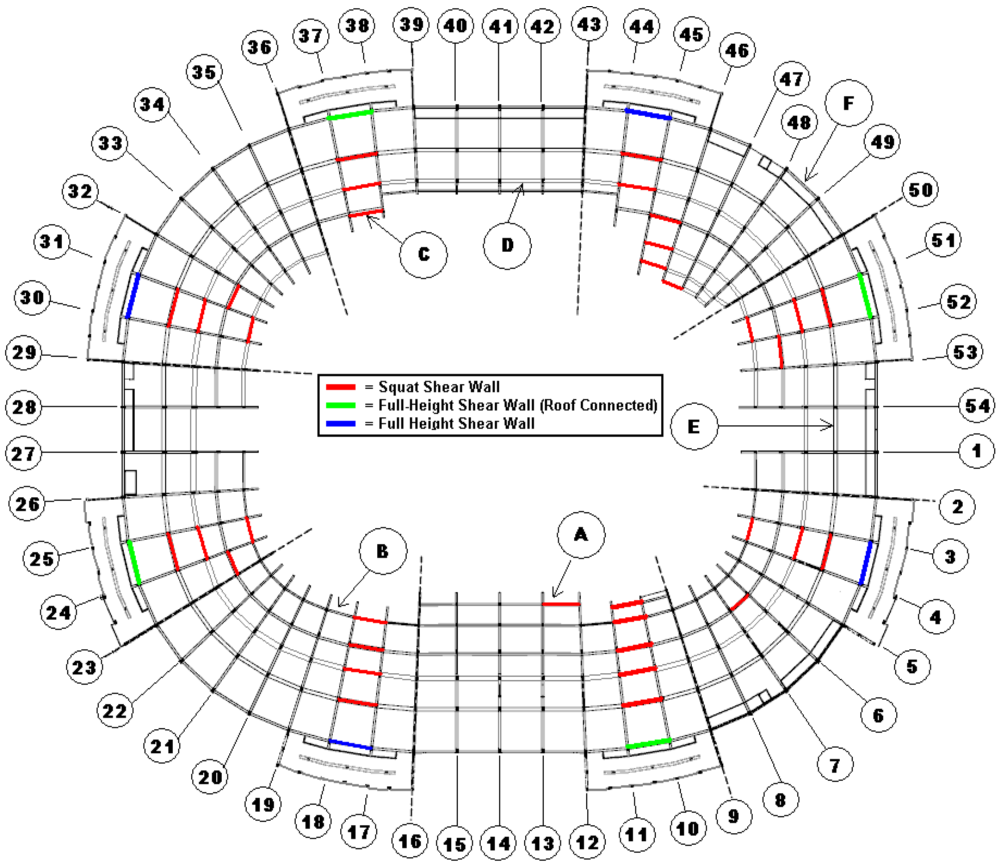

Figure 2. Plan view of BC Place Stadium, the green lines indicate full-height walls that are connected to the roof, the blue lines denote full-height walls not connected to the roof, and the shorter, squat walls are denoted in red. The eight ramp structures at the periphery of the building are self-supported and separate from the main bowl structure but do share common footings. Note the bowl structure is partitioned into eight segments and the expansion joints are noted by the dashed lines.

The reinforcing of the existing lateral load resisting elements is well detailed, suggesting that the frames will behave in a ductile manner. The frame-wall arrangement adds complexity in analyzing the structure, and the segmented bowl sections create the potential for pounding between bowl segments.

In order to control this drift and mitigate pounding soil anchors at the base of the shear walls are required, however, anchoring the shear wall foundations affect the dynamic characteristics of the structure as the anchored walls exhibit rigid response increasing the dynamic shear on these elements.

3. DYNAMIC ANALYSIS

Response Spectrum Analysis (RSA) were conducted on a full three-dimensional model of BC Place Stadium based on spectral acceleration values established in accordance with British Columbia Building Code (BCBC 2006) for a “Site Class C” soil classification with an importance factor, I_E, of 1.3. The RSA linear elastic method provided the design team with anticipated seismic demand in the structure by accounting for the stiffness of the building and its mass distribution (For concrete columns and shear walls the effective modulus of elasticity of concrete was considered as 70% of the computed value while for the beams the modulus of elasticity was considered as 40% of the computed value, as per Canadian concrete code CSA A23.1-04). The RSA analysis also confirmed that the dynamic characteristics of the roof are not sensitive to stiffness/mass changes in the bowl structure below (e.g. less than 5% change in fundamental period of roof structure considering cracked on uncracked section properties for concrete frames/walls for the bowl structure). A series of soil springs produced by the geotechnical consultant was introduced into the structural model to account for the structure boundary condition at the base.

Bounding the seismic demand on the structure was achieved using several modal combination methods. Modal combination methods that accounted for rigid mode response of the structure produced elastic base shears that were commensurate with the code prescribed base shears assuming the walls are fixed to the ground. The shear walls in most locations are squat or nearly squat in their proportions. The analysis generated shear demand on these walls that exceeded their sliding resistance and overturning moments resulting in significant uplift. In effect, the loads attracted in the (fixed-base) squat walls were quite large, and more advanced analysis was required as the squat walls were subject to foundation rocking/sliding.

For rocking foundations about 80% of total mass is participating in a period range above 0.5 seconds while for the fixed base structure the total mass participation for periods above 0.5 seconds is about 60%. The higher period associated with foundation rocking considerably reduced the anticipated peak seismic demand on the building (See Table 2).

Table 2: (Elastic) Base Shears from Multiple Modal Combination Methods and Shear Wall Boundary Conditions

		Walls Fixed to Ground	Assumed Foundation Rocking
	% Base Shear (BCBC 2006)	100%	100%
Dynamic Analysis: Modal Combination Method	GMC (Elastic – Not Scaled)	630,000kN	427,000kN
	CQC (Elastic – Not Scaled)	290,000kN	250,000kN
BCBC 2006 Static (Elastic) Base Shear		550,000kN	550,000kN

It is noted that the ‘conventional’ approach for seismic upgrades would be to beef up the wall webs to increase shear capacity and subsequently increase footing sizes to make these walls ductile; this, however, negatively impacts the remainder of the building as much higher design forces would have to be accounted for due to short period response of the building during a seismic event. To upgrade the footings soil anchors would be required to keep the drifts down, prevent foundation uplift, and pounding between building elements. While soil anchors might have relieved drift concerns and would ensure that the walls would be ductile (under much higher base shears); this approach makes the building much stiffer, attracting additional seismic loads not only to the primary elements but to all of the infill components and equipment therein. Anchoring the shear walls would be problematic not only for the sake of expensive excavation and installation given confined spaces, but it would also cause great disruption in the building as many mechanical and electrical rooms would have had to be relocated in and around the shear walls at ground level. This further emphasized the need for an innovative approach to complement the existing building's dynamic characteristics and minimize impact on the existing buildings systems.

Acknowledging the negative impacts of a conventional strengthen-and-anchor system, the design team focused on soil-structure interaction to attain a more comprehensive understanding of inter-storey drift associated with foundation rocking and the impact on the concrete sway frames in the transverse direction as they would be subjected to higher loads than would be anticipated under a fixed-base design. In order to verify the response of the building with unanchored foundations, detailed non-linear push-over and response-history analyses were undertaken to verify the story drifts, load sharing between the shear walls and sway frames out-of-plane, and an overall capacity of the entire system when accounting for plastic soil deformation compared with results obtained from a fixed-base, linear response spectrum analysis. The push-over analysis was a critical component of the seismic upgrade project, as it was able to verify that the rocking foundation approach was viable, and efforts could be focused in providing supplemental damping and energy dissipation mechanism in a form of viscous dampers; constructability and their incorporation into the project schedule in lieu of anchored foundation and upgrading shear walls was also considered.

4. NONLINEAR STATIC (PUSHOVER) ANALYSIS

Pushover analysis provides a simple method of directly evaluating nonlinear response of a structure at different levels of lateral displacements, ranging from initial elastic response through development of a failure mechanism (it is noted that the intent of pushover analysis was not to push the representative frames to a pre-set level of displacement target computed, as per formula in FEMA 356 document, but rather to establish the load-displacement response of the frame till mechanism occurred or numerical instability stopped the analysis). The sequence of yielding in beams and columns including foundation uplift was recorded together with the axial/shear/overturning moment demand on the circumferential shear walls.

The commercially available computer program SAP2000 was used to carry out the pushover analysis. The pushover procedures prescribed in the ATC-40 and FEMA-356 documents are fully integrated into the SAP2000 program. The pushover analysis was carried out for a typical radial and circumferential frame of the BC Place building, as shown in Fig. 3 below. Both models include the footings for columns and shear walls. The vertical soil springs representing soil properties under each footing were provided by the geotechnical consultant.

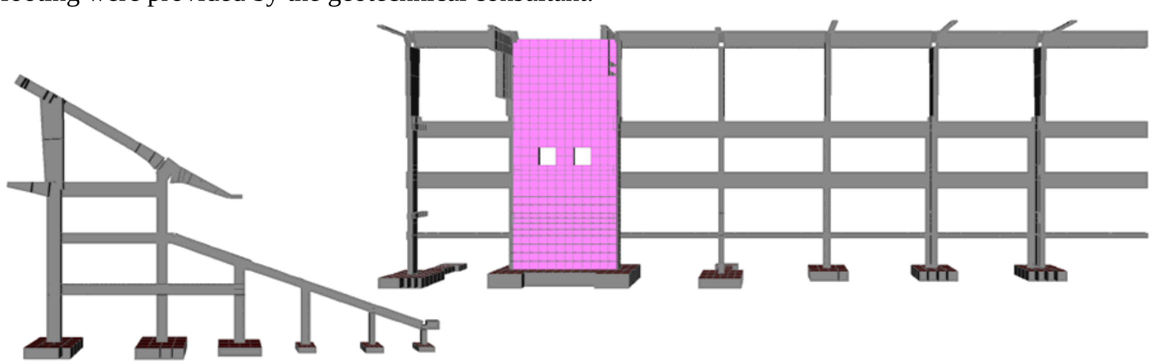

Figure 3: Elevation views of the BC Place radial (left) and circumferential (right) frames

Input parameters for the load-deformation response of the structural elements were adopted from FEMA 356. A strain-hardening slope of 2% of the elastic slope was considered for the post-yield load-deformation response of the frame elements. The interface of the frames and ground was modeled as a series of discrete spring elements depending on the elements of the soil underneath each support location. It is noted that the beam-column joints and the shear behaviour of the frames and walls were assumed to behave in a linear manner. This was later checked for representative elements to ensure the assumption of linearity is valid. All pushover analysis cases started after the application of the dead load case including 50% of the live load.

Fig. 4 shows the displaced shape of the circumferential frame at the end of pushover analysis together with the load-displacement plots for various loading patterns. The color spectrum at the bottom of the figure indicates the extent of nonlinear action. The purple color (far left) indicates start of nonlinear behaviour, the dark blue color indicates limited yielding in the region for immediate occupancy, the light blue is the life safety zone, and the green color is the collapse prevention zone while the yellow color is near collapse region.

The first yielding occurred in the beams adjacent to the shear wall at levels 3 and 4 at a lateral displacement of about 45 mm. Subsequently, the beams in levels 2 and 5 adjacent to the shear wall developed plastic hinging near the ends at a lateral displacement of about 70 mm at the fifth level. At a lateral displacement of about 120 to 140 mm the first sign of column yielding was observed at the base of 5th and 4th level columns adjacent to the shear wall. The analysis terminated at a lateral displacement of about 250 mm where the 2nd level beams exceeded the collapse prevention criterion.

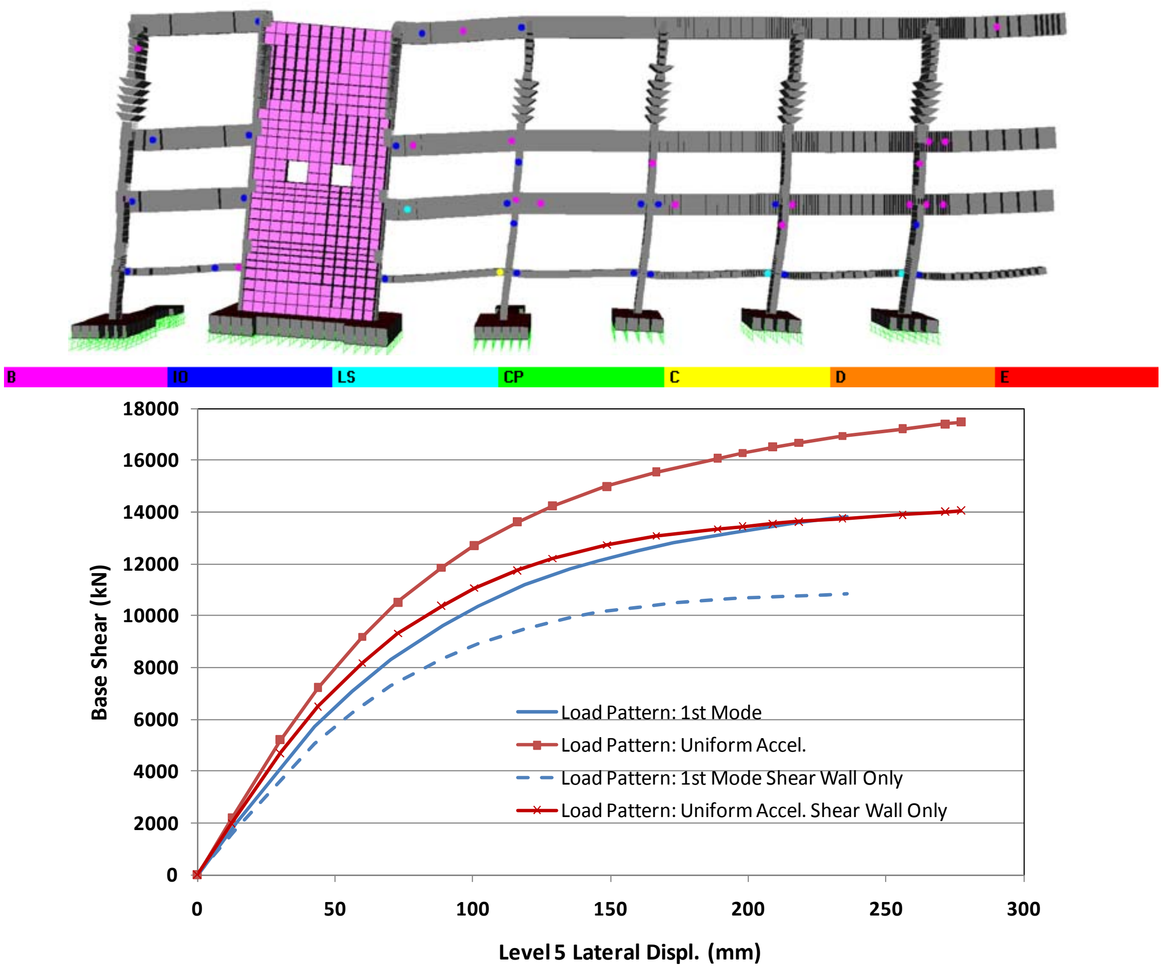

Figure 4: Extent of nonlinearity and plastic hinge formation at the end of pushover analysis (top) and load-displacement response of a circumferential frame (bottom)

The maximum uplift and downward displacements for the footing under shear wall were about 40 mm and 85 mm, respectively, for 1st mode loading and 55 mm and 95 mm for uniform acceleration loading corresponding to a bearing pressure of about 900 to 1000 kPa. The spread footings under columns did not experience uplift with a maximum downward displacement of about 40 to 50 mm.

The maximum axial, shear and overturning moment demand for the shear wall are about 31000 kN in compression, 14000 kN and 190,000 kN-m, respectively, for uniform acceleration loading pattern. It can be observed from the load-displacement plot that the shear wall attracts 80% of the total base shear while the remaining input shear demand is distributed among the five columns.

To minimize the displaced profile of the frames in the circumferential direction and to dissipate more energy during a seismic event the application of viscous dampers placed in the gap between the eight segments of the bowl structure and the gaps between the ramp walls and frames was considered. The selection of dampers was based on utilizing the gap between bowl segments and between bowl frames and ramps at various levels. The presence of the dampers links the sixteen structures of the base building (eight bowl segments and eight ramps) together during a seismic event, adding significant redundancy between individual building segments in a manner that did not negatively alter the period of the structure.

5. NONLINEAR DYNAMIC ANALYSIS

A comprehensive Probabilistic Seismic Hazard Analysis was completed for the purpose of earthquake record selection and scaling and carrying out nonlinear response history analysis. The ground motions were to be compatible with the design spectrum for the structure in the period range of interest, 0.5-1.0s, and to incorporate the effects of soil conditions at various locations on the site. The design spectrum corresponds to the uniform hazard spectrum (UHS) for Vancouver on Site Class C soils with a probability of exceedance of 2% in 50 years. The effects of local soil conditions on design ground motions were included by propagating the Site Class C motions through the local soils by site response analysis using the equivalent linear computer program SHAKE91 (modified from Schnabel and Seed 1972).

A 3D computer model of the building was developed using a commercially available software SAP2000 including the foundation elements modelled as thick SHELL elements. The boundary condition at the base of footings included vertical nonlinear soil springs provided by the geotechnical consultant at various locations. The horizontal springs were equivalent linear springs. Area GAP elements with nonlinear compression properties were provided for the foundation elements with a very small tension linear properties.

The effect of nonlinear viscous dampers ($F = CV^{\alpha}$) was included using NLINK elements in SAP2000 based on the information regarding stiffness and damping coefficients obtained from damper manufacturer. The properties used for a 2100 kN capacity damper are C=2500 kN.s/m, α=0.5 or C=2800 kN.s/m and α =0.4. Distance between Clevis Plates are 32mm + 38mm + 32mm = 102mm total, total width of damper + shims = 67mm (bearing width) + 14mm (shim) + 14mm (shim) = 95mm, extra (Play) for construction tolerance = 102mm – 95mm = 7mm.

Fig. 5 presents a 3D view of the computer model of BC Place stadium including a blow-up of the location of dampers between bowl segments (total of eight dampers at different levels at each expansion joint plane) and between bowl radial frames and ramp structure (four dampers at level 4 between ramp structure and radial frames).

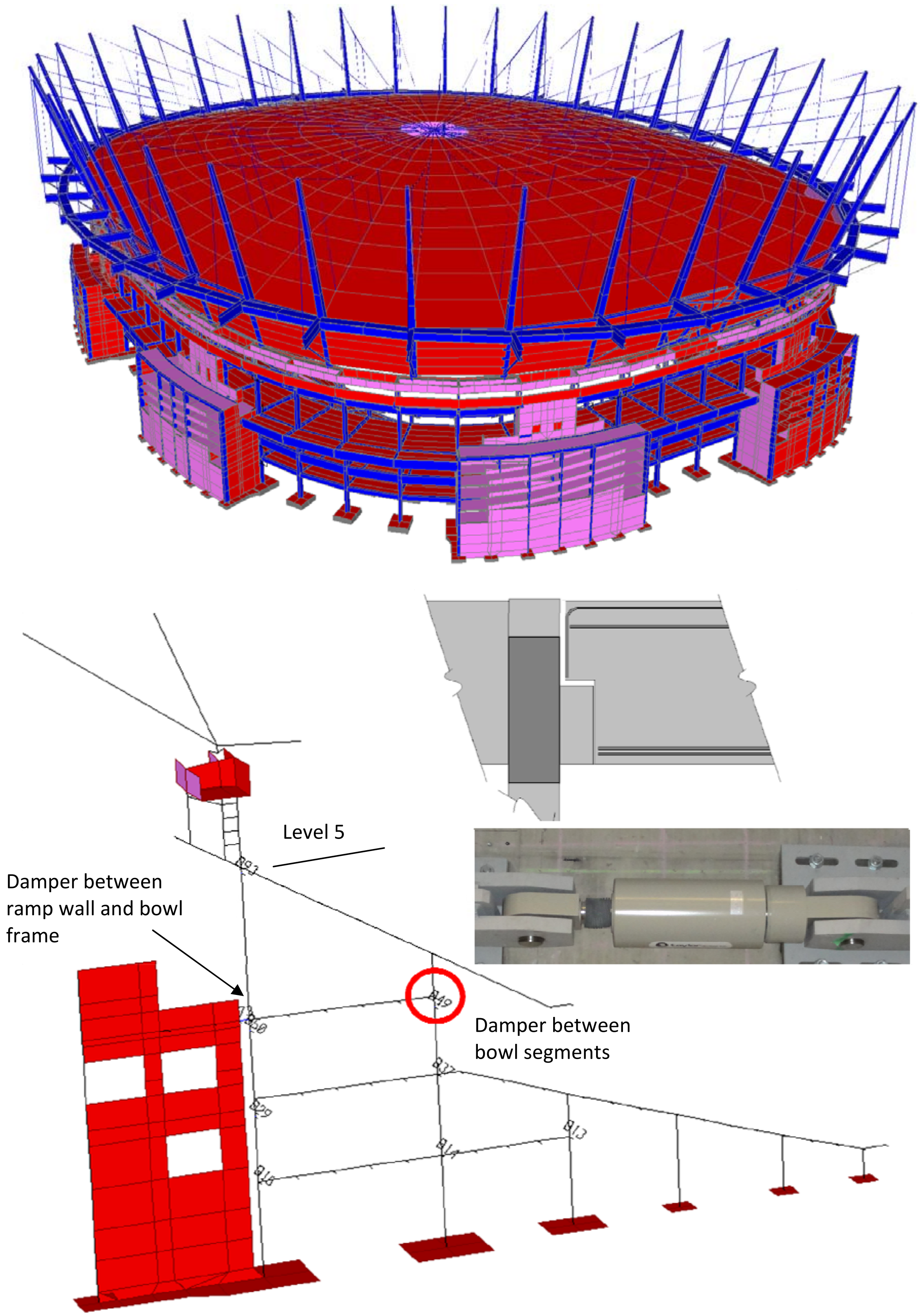

Figure 5: A 3D View of BC Place Stadium Computer Model including new retractable roof structure and periphery access ramp walls (top) and blow-up detail of damper locations including existing gaps

The results of time-history analysis confirmed the rocking response of the building together with a significant energy absorption at the boundary of bowl segments and bowl and ramp structures by the presence of dampers. Fig. 6 shows typical load-displacement and velocity response-history of dampers. The maximum displacement demand was computed as about 60 mm while the force demand was close to the capacity of the dampers. The maximum velocity demand of 0.8 m/s was computed.

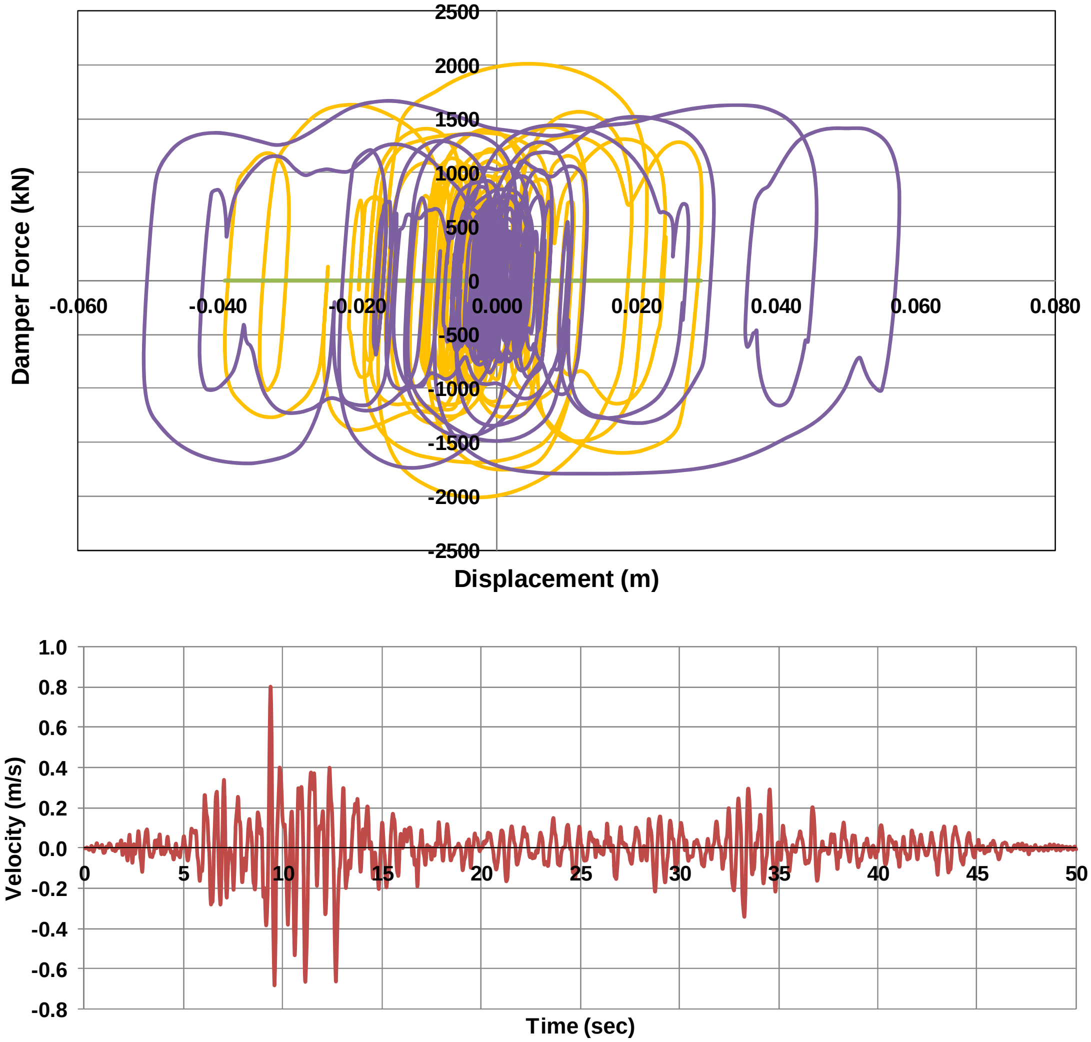

Figure 6: Load-displacement and velocity response history of typical viscous damper(s) at level 5

It is noteworthy that this exceedingly complicated analysis and review of multiple time-histories was conducted in a timely manner to optimize the damping properties of the devices in addition to reviewing the demands on the structure and soil below simultaneously. While the detailed analysis was underway, a broader analysis was also carried out concurrently in order to devise a range of damper characteristics anticipated for the purpose of tender and to initiate procurement and construction of the devices while the final details of the dampers were being completed; this higher-level parallel approach was a key to the success of project's completion on schedule.

6. CONCLUSIONS

The results indicate the significance of soil-structure interaction and the effect of foundation rocking and nominal soil nonlinearity on the overall response of the building (e.g. reducing the base shear demand by about 60 to 70% of that assuming a fixed base). This behaviour would also result in increased displacement response of the superstructure and permanent foundation displacements. Further response history analysis verified the results of pushover and response spectrum analyses by considering a linked bowl structure including ramps using viscous dampers to mitigate the pounding between bowl segments that would largely result from the increased displacement associated with rocking foundations. Linking the bowl would have the added benefit of more evenly distributing lateral forces amongst the frame and wall elements throughout the building.

ACKNOWLEDGMENT

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